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SANTA CLARA VALLEY WATER DISTRICT

HYDROLOGY

DOCUMENTATION FOR THE NORTHWEST FLOOD CONTROL ZONE

Prepared by

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INTRODUCTION

This report summarizes the work completed on the hydrology within the Santa Clara Valley Water District's Northwest Flood Control Zone. Presented in the report are recommended design flows for San Francisquito, Matadero, Barron, Adobe, Stevens, and Permanente Creeks, and each of their respective tributaries. They are divided into several hydrologic studies each containing a table of contents, a general information about the physical and hydro-meteorological features of the basin, and a discussion of the procedures followed. This last item includes the analysis of recorded data, as well as recorded and reconstituted data. It also includes a comparative analysis of the regional and site specific statistics.

The choice of the final design flows is based on findings from the above results. It is the intent of this report, as well as other similar reports, to present a documentation of the hydrologic assumptions made together with the steps taken in the mathematical calculations. Each creek was originally treated separately. Inasmuch as the study has been combined in terms of flood control zones, some redundancy is to be expected.

The following is a list of the tributaries analyzed in these studies:

San Francisquito Creek Hydrology
  - Bear Creek
  - Los Trancos Creek
  - San Francisquito Creek

Matadero-Barron-Adobe Creeks Palo Alto Flood Basin Hydrology
  - Adobe Creek
  - Barron Creek
  - Deer Creek
  - Matadero Creek
  - Palo Alto Flood Basin

Stevens and Permanente Creeks Hydrology
  - Hale Creek
  - Permanente Creek
  - Permanente Diversion
  - Stevens Creek

SUMMARY OF METHODS AND PROCEDURES

General

The District has established its flood protection criterion for watershed areas greater than four square miles based on flows which has a 1% chance of being equaled or exceeded in any one year. For watershed areas between one and four square miles the criterion is the 2% flood and for watershed areas between 360 acres and one square mile it is the 4% flood.

The accompanying reports represent the 1977-78 update of the design flood flows for use in the design of flood control facilities in Santa Clara County. They present documentation for the calculation of the 1% and 10% design flood flows. Other design values (2% and 4%) are not presented herein; they could be obtained upon request. These values were calculated using the District's methods and procedures as augmented and modified by the recommendations of the panel of expert hydrologists* convened to review the methods in 1976. It was assumed in the derivation of these design flows that the channel system is free from major constrictions or significant flood storage (reservoirs are handled separately) and that the floodwaters at the design level are completely contained within channel banks or relatively narrow floodway limits.

The District's method of determining the design peaks and volumes of floods utilized all available sources of information to derive the most accurate values possible. In general, this method began with the use of Regional Regression Equations to compute peaks and volumes. These peaks and volumes were compared with the results of an analysis of reliable recorded streamflow data when such exist. The values were also compared with the results of a rainfall-runoff model before the final "design flow" classification is given. Where design flows are presently affected by reservoirs or by urbanization, special analyses accounting for their effects were performed. Finally, the design flows were compared with the results of other studies and with previously determined design flows. The results of such comparison are useful to further define the design values.

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*The panel members were Leo Beard, Dave Dawdy, Neil Grigg, Douglas L. James and Vujica Yevjevich. Their report, "Review of Basic Hydrology Methodology for Flood Control" Santa Clara Valley Water District, 1976, is available from the District library.

Hydrographs were derived utilizing the Corps of Engineers' standard project storm and the Clark Synthetic Unit Hydrograph technique. The peaks and volumes of these hydrographs were then adjusted to agree with those values derived from regional or local statistics. The flood hydrographs obtained this way are considered representative for use in the design of most flood control facilities. However, where flood traps (small volume peak skimmers) are being considered, a sensitivity analysis of the design relative to hydrograph shape should be conducted.

Following is a more detailed disucssion of these methods. Full documentation of the analyis of each stream is available in separate reports.

Regional Regression Analysis

The backbone of the District's hydrology is the application of the Regional Regression Equations (1976) for the determination of flood peaks and volumes. These equations were obtained by analyzing recorded streamflow data (peak flows and their respective volumes) from stations within and outside the District. The data were correlated, expanded, and the resultant statistics adjusted to obtain the dependent variables (mean, standard deviation, and 1% flow). These statistics were then regressed to the physical characteristics of their respective basins (independent variables) by means of a mathematical process called Multiple Linear Regression. The regression equations chosen had minimal standard errors of estimates and maximum determination coefficients. These equations are applicable to all unurbanized locations within the District to generate statistics for the determination of design flood peaks and volumes.

Gaged and Ungaged Locations

At gaged locations the statistics from the regression equations were compared with the site-specific statistics obtained from recorded data when such data were usable. The statistics from recorded data were obtained using the methods presented in the Water Resources Council Bulletin #17. If there were significant differences between the regional and site-specific results, then new statistics (weighted with respect to their respective standard errors) were calculated. These statistics, together with a Pearson Type III distribution a -0.6 regional skew coefficient or a weighted one, were used to determine the frequency regime.

At ungaged and rural locations, the results of the regression equations are considered to provide the best available information and, hence, are to be used for analysis and design.

Techniques Used for Developing Flood Hydrographs

Pattern runoff hydrographs are obtained from rainfall using Clark's Synthetic Unit Hydrograph method (as developed by the Corps of Engineers' HEC-1 computer program). This rainfall runoff transformation involves the determination of loss rates, synthetic unit hydrograph parameters, spatial distribution of rainfall over the watershed and temporal distributions of rainfall rates within one storm. The first two items are usually obtained from projections of optimized results of recorded rainfall and its respective runoff data. The averaging of rainfall over the basin is usually done by the regular Theissen weighing operations. The temporal distribution is determined generally by using the Corps of Engineers' Standard Project Storm for Northern California.

If the peaks and volumes of the pattern hydrographs obtained are drastically different from the regional or site-specific results, then the reasons were investigated and, if justified, further adjustments were made. In this study, the rainfall-runoff-generated peaks and volumes are found to generally agree with regional or statistical values when average rainfall and area configurations were present. To avoid the uncertainties inherent in the rainfall-runoff transformation, the peaks and volumes generated from this transformation are balanced by regional values.

The balanced hydrographs obtained above are then combined with other hydrographs and routed downstream. The routing was done using linear methods (e.g., Muskingum by computing k when x = 0) if routing through channel system, or nonlinear methods (modified Pulse) if significant storage is available.

Rainfall distribution, time of concentration and the various routing operations are the items that define the shape of pattern hydrographs, with rainfall distribution being the foremost. The effect of the shape of these hydrographs, given constant peaks and volumes, may become significant when flow trapping is being considered as a flood protection measure. An in-depth analysis of the shape was not attempted as a part of this work.

Regulated Watersheds

In regulated watersheds, regulated by reservoirs, the design flows are assumed as inflows to the reservoirs. The initial storage levels of the reservoirs that coincide with the design flows of certain frequencies are computed from the stage-duration frequency (S.D.F.) analysis of daily recorded reservoir volumes. The Corps of Engineers' Coincidental Frequency Analysis (discussed in Leo Beard's "Statistical Methods") procedures were used to solve this compounded probability problem. At most reservoirs, the S.D.F. curves after 1968 were used because, since that time, the reservoirs have retained more water in storage longer to enhance recreation.

Urban Hydrology

The above procedures are applicable mainly to rural watersheds. For urban watersheds, the methodology changes and the regional regression equations do not apply.

Urban hydrology deals with the effects due to two major changes in the watershed, namely:

1. Increased imperviousness
2. Channelization

The increase in imperviousness is caused by rooftops, streets, parking lots, driveways, and other man-made ground surface cover. Channelization includes the flow of water in gutters, storm drains, improved channels, etc. An increase in the imperviousness tends to reduce the loss rates and increase the runoff volumes and peaks, while channelization may either increase or decrease the peaks. It depends on the magnitude of the flood and the storage-discharge capacity of the various channels or storm drain systems that carry the flood.

To determine the effect of increased imperviousness, the urban area of every watershed was divided into subareas tributary to the major storm drains. These subareas are then transformed into equivalent rectangular subareas where the length is equal to the length of an equivalent street. The ratio of imperviousness (which is a measure of land use) is then used to subdivide this equivalent subarea into two sub-subareas of pervious and impervious parts.

In order to calculate the separate inflow hydrographs from the two sub-subareas, the unit hydrograph procedure was employed using separate times of concentration for both the pervious and impervious parts. The routing coefficient is calculated from the length and width of these areas. The loss rates, which are the third set of parameters needed for the computation of the one percent flood hydrograph, are assumed to be 30 percent for the pervious part and five percent for the impervious part. Since most of the floodwater from urban areas is conveyed to flood control channels by the storm drain system, these hydrographs are routed through the storm drain system utilizing the modified Pulse routing method. This routing requires storage-discharge curves for the system of streets, gutters, manholes, and storm drains. Two actual urban areas, one of low-density residential and another of high-density residential and commercial, were used to develop equivalent generalized storage facilities and storage-discharge curves. The resultant curves were transformed to reflect unit areas, and then compared with each other. Since the two curves are very similar, an average smooth curve was chosen and used.

Because most of the urban area in Santa Clara Valley is in the flatland and similar storm drain design criteria were used throughout, it is assumed that the unit storage-discharge curve that was obtained from the above study is applicable to all the urban areas in the Valley.

The above method is detailed in a paper presented before the National Symposium on Urban Hydrology in 1976.

Upper 80 Percent Confidence Limits

As a District policy, the upper 80 percent confidence value for the 1% design flow rate is used as one input to define freeboard criteria. It is defined as the value at which there is a 90 percent chance that the design flow is equal to less than the given value. It relates to the standard errors associated with the statistical inferences of information about populations from very limited samples. These values are calculated using the Non-Central Student "t" Distribution as shown in the Water Resources Council Bulletin #17.

Previous Studies and Coordinated Results

As a part of the design flood flow determination procedure, all available hydrologic analyses done on any watershed (within the County) were reviewed and considered.

These previous analyses were sometimes the result of an in-house effort and other times the result of efforts of other agencies.

Because of the nature of random events, especially in the case of small samples, the results obtained from previous analysis were not necessarily congruent with the new values. Therefore, an in-depth search of the adequacy of the various results and methods involved was done.

As a result of this study, some of the adopted values were supported not only by District-developed methods and techniques but also by those of the Corps of Engineers and others.

Further Information

The attached information is only a summary report of that available. Additional information, available upon request from the Hydrologic Library, is listed below:

1. Hydrographs, either plotted or numerical, at all locations.
2. Runoff from intermediate areas such as urban runoff.
3. Recorded streamflow data at most gaged locations.
4. Stochastically estimated data at most gaged stations.
5. A full spectrum of statistical analysis of the above data which include the investigation of outliers, the incorporation of historic flows, and the adjustment of statistics and probabilities.
6. Statistical analysis of rainfall data on all gaged locations.
7. Complete descriptions and documentation of computations done for other streams in the County.
8. Other supporting documents of the methods like the 1976 Panel of Experts Report, the 1976 Regional Regression Study, Paper on Urban Hydrology and Reports by Zone, discussing the hydrology in detail.

ACKNOWLEDGMENTS

E. S. Watson, Division Engineer, was very instrumental in directing the work and reviewing this report.

Randy Talley compiled the data and did the initial drafting of the Matadero, Adobe and Barron Creeks part of this report.

SAN FRANCISQUITO CREEK HYDROLOGY

DETERMINATION OF DESIGN FLOWS

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GENERAL INFORMATION

San Francisquito Creek drains the eastern part of Santa Cruz Mountains between Kings Mountain and Russian Ridge. The watershed (Figure 1-1) is mushroom in shape, with the upper portion stretching along the San Andreas fault in the north-south direction. The lower part of the watershed, easterly of Highway 280, is narrow and mostly urban.

Tributaries to San Francisquito Creek are:

1. Bear Creek

2. Alambique, Martin, Sausal and Corte Madera Creeks. These creeks drain into Lake Searsville and are located in San Mateo County.

3. Los Trancos - This creek forms the northern boundary of Santa Clara County. It joins San Francisquito Creek at a location downstream of Searsville Lake or 1/2 mile north of Highway 280.

The average slopes of these creeks range from 100 to 160 feet/mile. The slope of the lower part of the creek or downstream of Alpine Road is almost a constant 50 feet/mile.

The existing capacity of the channel is 12,000 c.f.s. near the foothills and ranges from 9,000 c.f.s. near Stanford to 6,000 c.f.s. downstream of El Camino Real.

The objectives of this report is to document the hydrology work done to determine design flows. It will discuss the use of regression equations and the analysis of recorded data. It reviews previous work done by various agencies and compares the results with the present ones.

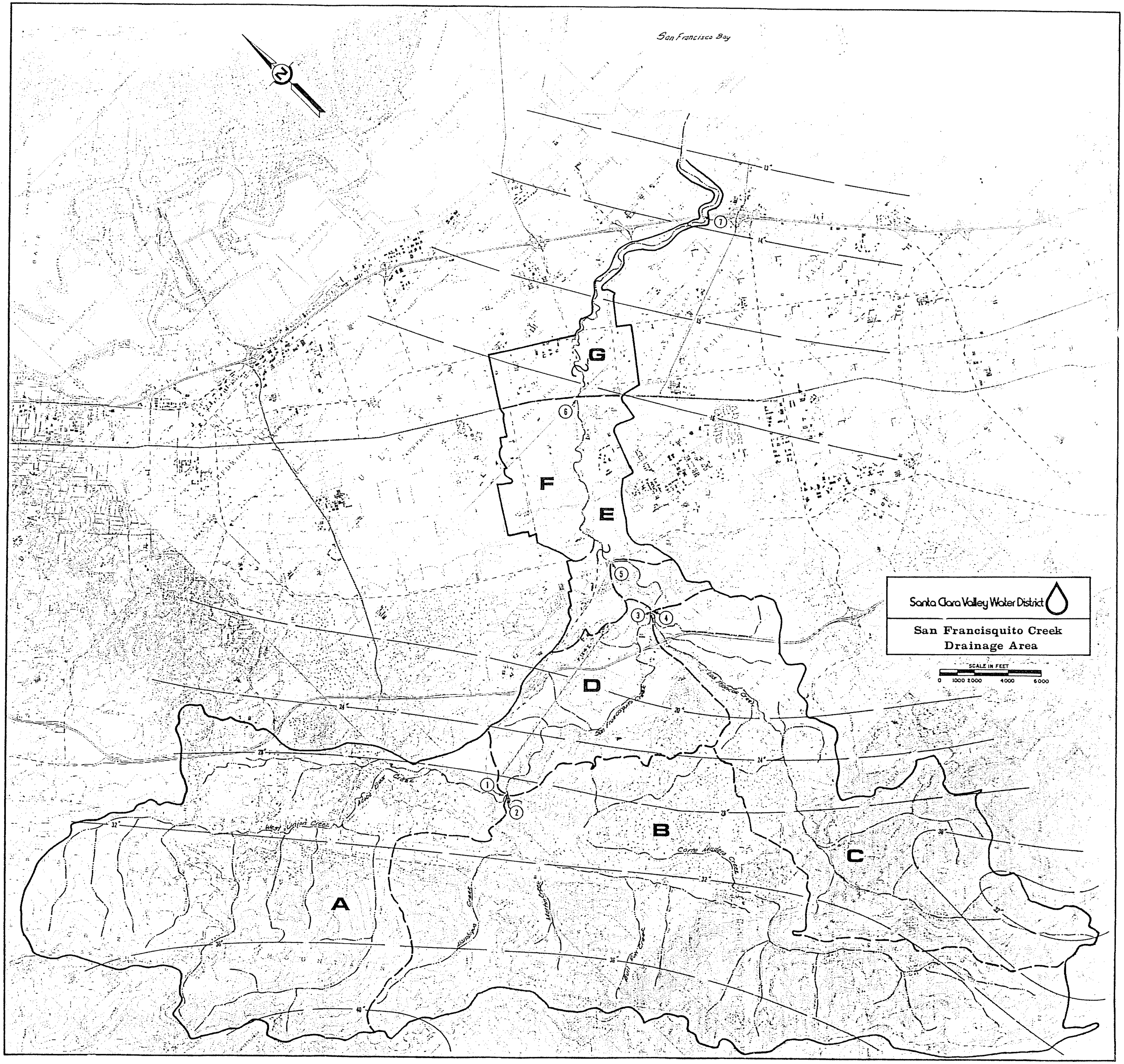
San Francisco Bay
N
Santa Clara Valley Water District
San Francisquito Creek Drainage Area
SCALE IN FEET
0
1000
2000
4000
6000
A
B
C
D
E
F
G
①
②
③
④
⑤
⑥
⑦
West Union Creek
San Francisquito Creek
Los Trancos Creek
Corte Madera Creek
Alambique Creek
Martin Creek
Bull Run Creek
13"
14"
15"
16"
20"
24"
28"
32"
36"
40"

Fig. 1-1

Climate and Hydrologic Characteristics of the Basins

The San Francisquito Basin has warm dry summers and mild wet winters. The mean annual precipitation ranges from 13 inches near the Bay to 40 inches near Skeggs Point in the Santa Cruz Mountains. The highest point in the watershed is on Borel Hill (Elevation 2,570). The distribution of vegetative cover follows the mean annual precipitation pattern. It ranges from forest cover on top of the Santa Cruz Mountains where rainfall is highest, to pasture land at lower elevations. The flat land is mainly urban with densities ranging from very low residential in the upper part to high density residential and commercial in the lower part.

The basin has one reliable U.S.G.S. streamflow gaging station located on San Francisquito Creek at Junipero Serra Boulevard (U.S.G.S. Station #1645). The reliable record of this station extends from 1932 to 1941 and from 1951 to the present, for a total of 33 years.

There are many rainfall stations that collectively represent the precipitation patterns on the watershed. These stations are listed below:

1. #73 - Palo Alto City Hall, N.W.S. Bucket
2. #63 - Mountain View Fire Station 8, Co-op, Recording
3. #53 - Maryknoll - S.C.V.W.D., Recording
4. #24 - Dahl Ranch, S.C.V.W.D., Recording
5. #94 - Saratoga Gap, Private, Storage
6. #96 - Searsville Lake, N.W.S., Storage

PREVIOUS STUDIES

San Francisquito Creek lies on the border line of two counties, Santa Clara and San Mateo, and the watershed extends into both counties. For this reason and because of the urban activities, as well as the academic activities in the area, the hydrology has been analyzed and reanalyzed by various agencies. These analyses are discussed briefly in this report.

Corps of Engineers' Study (1971)

Based on the information from Lake Searsville, the Corps of Engineers estimated 79 years of streamflow recorded (1883-1971) at the gaged location. This data was fitted with a Pearson Type III Distribution with a resultant 1% design flow of 9400 cfs (Reference 6). Unfortunately, the backup material for that reference has been lost, according to the San Francisco District, Corps of Engineers. Therefore, no further information is available as to the development of this design value.

This same data (79 years) was used in our comparative analysis as will be discussed later.

Stanford University Study (1967)

As part of an advanced engineering degree dissertation, J. R. Vincent of Stanford studies San Francisquito Creek (Reference 7). The study included the statistical analysis of the Corps of Engineers' data, mentioned above, by 3 different approaches; Beard's statistical, a graphical and Gumbel. Then a new set of data was generated by simulation (1911-1967) utilizing the Stanford watershed model, and the same three statistical methods were used in the frequency analysis to generate six alternatives.

The results from these six different approaches were then compared with each other. The final choice of the frequency curve was then based on Gumbel's distribution as fitted to simulated data (1% value = 6400 cfs). This curve was then shifted upward to approach the value obtained from fitting Beard's distribution (1% - 7100 cfs) and a 1% design flow equal to 7000 cfs was obtained.

Santa Clara Valley Water District Study

In 1977 the District conducted a study of the hydrology of San Francisquito Creek. The method included the utilization of recorded data from U.S.G.S. Station No. 1645 together with stochastically estimated data (Reference 4) and rainfall information. This information was all used to augment the results obtained from the regression equations.

The specific description of data analysis and other procedures are discussed below:

Analysis of Recorded Data

The thirty-five years of recorded peak flows (two missing years out of those were estimated as a part of the S.C.V.W.D. Regional Regression Analysis - Reference 4) were analyzed using the Water Resources Council (WRC - Reference 5) frequency analysis procedure. In this set of data, the high flow of 5560 cfs occurred on December 22, 1956 while the low flow of 12 cfs occurred on November 26, 1960. The 12 cfs proved to be a low outlier (Reference 5). Therefore, it was eliminated from the statistical calculations and the adjustment for low outlier was made.

Using a -0.6 regional skew coefficient, the 1% design flow was then computed equal to 6770 cfs. On the other hand, if the 12 cfs were arbitrarily increased to 14 cfs, the low outlier would be eliminated , and the design flow would become 8,953 cfs.

The sensitivity of this situation demanded further investigation into the accuracy of this particular low flow value of 12 cfs. John Lamirinos of the U.S.G.S., Palo Alto, was contacted and his answer confirmed that the 12 cfs is indeed the best value.

Stochastically Generated Data

As a part of the regional regression analysis (Reference-4), 61 years of stochastically generated data were statistically analyzed. The analysis yielded a 1% design flow of 10,000 cfs.

Utilizing the three types of data (1) estimated by the Corps of Engineers, (2) simulated by Stanford University or (3) estimated by the District by using the new skew factor and distribution functions, the three sets of data yield similar results (Q 1% approximately equal to 10,000 cfs). These results are summarized on Table 1-1 and the complete statistical analysis is available from the District's Hydrologist.

Before deciding on which value to use (either approximately 10,000 cfs or approximately 7,000 cfs), further analysis of the results was conducted. This analysis included information from rainfall, historic and stochastic data and from weighing of statistics as recommended in Reference 10.

TABLE 1-1

COMPARISON OF RESULTS OF STATISTICAL ANALYSIS
Using Pearson Type III Frequency Distribution

	S.C.V.W.D.[1]	C. of E.[2]	Stanford[3]
N	1915-1975 = 61 years	(1894-1967) = 75 years	(1911-1967) = 57
Ne, equivalent years	53.5	-	-
Arithmatic Mean	1728. c.f.s.	1789.	1690.
Mean of logs of values	3.02515	3.06726	3.01589
Standard Deviation of logs	.5274	.4821	.52804
Calculated Skew	-1.2550	-1.3606	-1.2071
10% Flood	3900 c.f.s.	3750 c.f.s.	3880 c.f.s.
1% Flood	5900 c.f.s.	5300 c.f.s.	6120 c.f.s.
Zero Skew (Normal) (K = 2.32635)	Q 1% = 18,086. c.f.s.	Q 1% = 15,618.	Q 1% = 17,765.
Regional Skew (-.6) (K = 1.88)	Q 1% = 10,390	Q 1% = 9410	Q 1% = 10,220

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1. Estimated flows of 1943 and 1945 were adjusted based on the Corps of Engineers and Stanford Studies.

2. Estimated zero flows for years, 1929 and 1931 were adjusted based on S.C.V.W.D. and Stanford studies.

3. Estimated flows for years, 1914 and 1921, which were ommitted in Reference 1, were adjusted based on S.C.V.W.D. and the Corps studies.

APPLICATION OF 1976 REGIONAL REGRESSION EQUATIONS

To further examine the statistical analysis of recorded data, and to find the required hydrologic information at locations different from the gaged location, District's 1976 regional regression study (Reference 4) was utilized for rural areas and District's urban hydrology method for urbanized areas. The bigger part of the San Francisquito watershed is rural area.

Table 1-2 shows the parameters used in this regional analysis, while Table 1-3 shows the results obtained from the application of the regression equations obtained from the above study. The 1% flow at the gage location is equal to 9860 cfs, which compares favorably with results from the statistical analysis of the recorded and estimated data. Yet, these regional results should be weighted with recorded data to employ the site-specific information.

TABLE 1-2

Basin Physical Parameters for Use in 1976 Regional Equations (Obtained from U.S.G.S., 7-1/2 min. quad sheets)

Name of Subwatershed	Area	A Sq. mi.	P in.	L mi.	Lca mi.	H/S (Ft/Ft)	LLca/S$^{\frac{1}{2}}$
Bear Creek @ (1)	A	12.24	34.0	6.6	3.4	280/.008	250
Corte Madera & others (2)	B	14.63	33.46	7.6	2.3	500/.012	160
San Fran. @ (4)	D	3.26	21.80	3.4	1.1	55/.004	59
Los Trancos @ (3)	C	7.25	27.50	6.6	2.6	480/.014	145
San Fran. @ (5)	D'	.13	18.00	.8	.4	.004	5
A+B		26.87	33.76	7.6	.70	.01	53
A+B+D		30.13	32.42	10.0	1.9	.009	200
A+B+D+D+C (U.S.G.S. Sta. #1645)		37.5	31.00				606.1
Stanford East of Creek	E	1.74	17	2	1	.004	30.

TABLE 1-3*

PEAK FLOWS & 24 HR VOLUMES AS OBTAINED FROM 1976 REGRESSION EQUATION

AREA	PEAK FLOWS				24-HR. VOLUME (c.f.s.)			
	M	σ	1%	10%	M	σ	1%	10%
A	2.80204	.4449	4349	2167	2.4594	.44454	1913	967
B	2.84602	.43818	4675	2354	2.51319	.43991	2123	1091
D	2.1701	.5055	1319	598	1.69104	.51478	440	202
C	2.5322	.47627	2677	1270	2.13147	.47956	1044	505
D'	1.11457	.48703	107	50	.404	.50934	22	10
A+B	3.0304	.41297	6409	3357	2.73818	.41916	3263	1731
A+B+D	3.0422	.44948	7712	3815	2.7529	.45034	3855	1944
A+B+D+D'+C	3.0825		9859	4615			4914	2375 (USGS #1645)
E	1.8496	.54656	753	320	1.30163	.56177	219	94

*Peak Regression Equations: $M = .148 + .68 \log A + 1.25 \log P$

$$\sigma = 1.65\ P^{-.45}\ LLs^{.05}$$

24-Hr. Volume Regression Equations: $M = .756 + .83 \log A + 1.51 \log P$

$$\sigma = 1.805\ P^{-.46}\ LLs^{.04}$$

where, M is the mean of the logs of flow
σ is the standard deviation of the logs

COMPARISON OF STATISTICAL AND REGIONAL RESULTS

It was shown that the statistical analysis for recorded data with a low outlier on San Francisquito produced the following results:

Mean of logs = 3.1388

σ = .3728

Q 1% = 6,770 c.f.s. with a regional skew coefficient of -0.6 and a -.627 adopted one, as obtained by weighting the regional skew with that of recorded data as per the W.R.C. Bulletin #17

This does not compare favorably with values obtained from the regional analysis (9860 cfs) and from estimated data (10,400 cfs). The final results were weighted to combine the information from regional and recorded values. The following procedure of weighting is followed:

It is known that the standard errors of the regional equations are as follows:

Standard error for the mean, $\varepsilon_m = \frac{\sigma}{\sqrt{N}} = .21$[1/]

Standard error for the Standard Deviation, $\varepsilon_\sigma = \frac{\sigma}{\sqrt{2N}} = .065$[1/]

The equivalent years of record (Ne), is therefore, equal to

$$Ne,\ m = \frac{\sigma^2}{\varepsilon_m^2} = \frac{(.4847)^2}{(.21)^2} = 5.3 \text{ years for the mean}$$

$$Ne,\ \sigma = \frac{\sigma^2}{2\varepsilon_\sigma^2} = \frac{(.4847)^2}{(.065)^2} = 27.8 \text{ years for the standard deviation}$$

[1/] Reference is made to the report (Development of Regional Regression Equation, #4).

These equivalent years of record are used in weighing the final results to include all the information in and around the area of interest.

The mean of the station's record is, therefore, equal to the weighted average of the mean obtained from statistics and the mean obtained from regional equations. In other words:

$$\text{Weighted Mean} = \frac{\text{M statistical} \times \text{N} + \text{M regional} \times \text{Ne.m}}{\text{N} + \text{Ne,m}}$$

$$= \frac{(3.1388 \times 35) + (3.08254 \times 5.3)}{40.3}$$

$$= 3.13140$$

Similarly, the variance of the station is,

$$\text{Weighted } \sigma^2 = \frac{\sigma^2 \text{ statistical} \times \text{N} + \sigma^2 \text{ regional} \times \text{Ne}, \sigma}{\text{N} + \text{Ne}, \sigma}$$

$$= \frac{.3728^2 \times 35 + .48473^2 \times 27.8}{62.8}$$

$$= 0.18147$$

and the standard deviation (σ)

$$= .42599$$

$$\log Q\ 1\% = 3.13140 + 1.88 \times .42599 = 3.9323$$

$$Q\ 1\% = 8500 \text{ c.f.s.}$$

The 1% flows of the regional equations as shown in Table 1-3 (9,860 cfs) for the 1% flood @ U.S.G.S. location) should be multiplied by 0.87 to include information inherent in the recorded data.

Similarly, the weighted 10% flow is computed by taking the weighted mean (3.1314) and standard deviation (.42599) together with a deviate of 1.200. The 10% flow is equal to 4400 c.f.s. which is 95% of the regional result of 4615 c.f.s. Figure 1-2 is a visual representation of the frequency curves as obtained from various methods. Table 1-4 shows the adjusted design flows at various locations.

September 1976

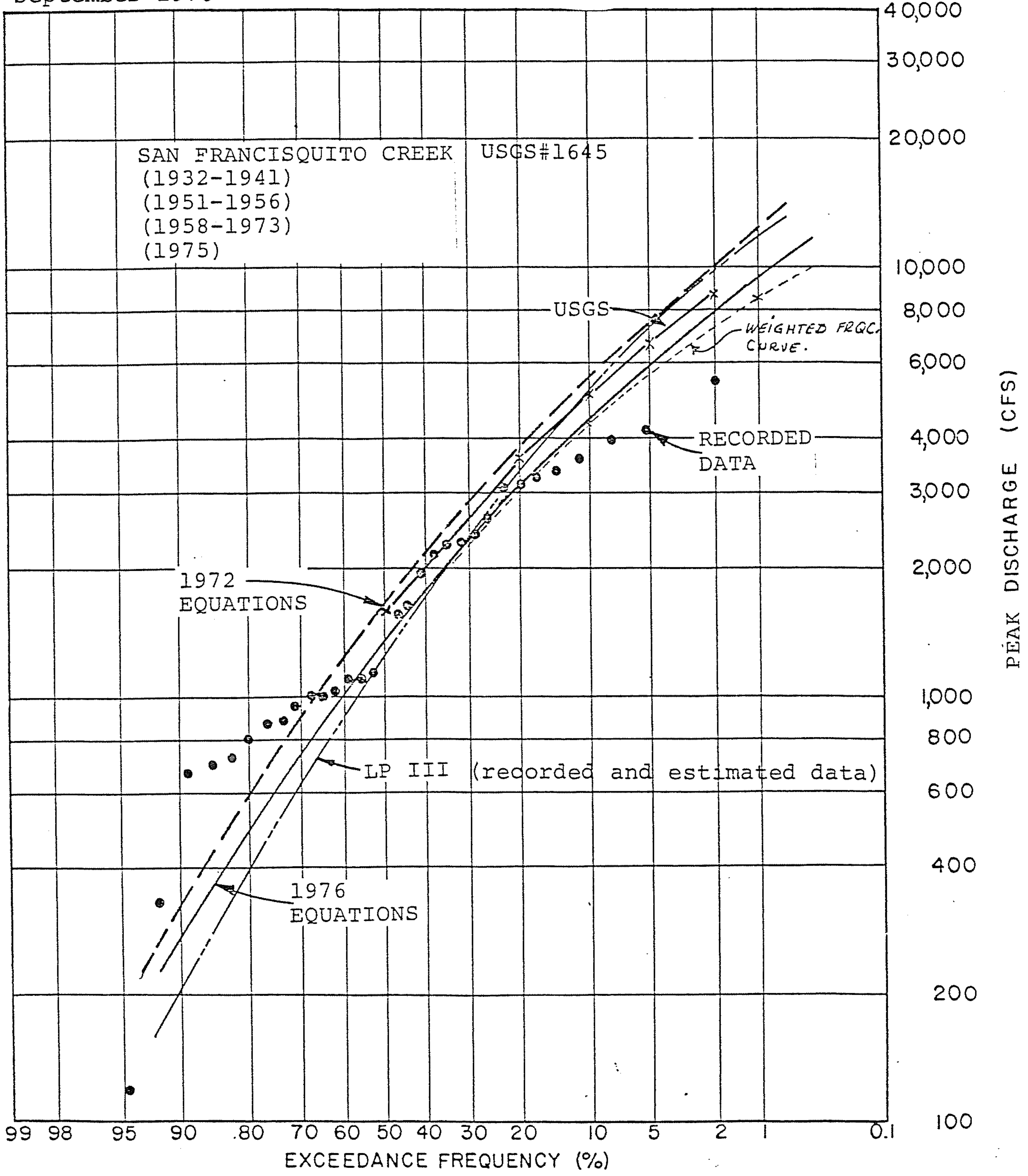

Fig. 1-2

TABLE 1-4

WEIGHTED DESIGN FLOOD FLOWS

Area	$Q_{1\%}$ c.f.s.	$Q_{10\%}$ c.f.s.
A	3780	2060
B	4070	2240
D	1150	570
C	2330	1200
D'	90	47
A+B	5600	3180
A+B+D	6700	3620
A+B+D+D'+C	8600	4380
E	650	300

RESULTS AND CONCLUSIONS

The following Tables 1-5 and 1-6 are the summary of the 1% and 10% design flows at various locations. The values in these tables should be rounded up to significant figures.

These design values are obtained from the application of the 1976 regional regression equations as adjusted with recorded and stochastical data (Table 1-4). Further checking was done against pattern hydrographs generated using a certain pattern of rainfall. Complete printout of these peak flows with their respective pattern and balanced hydrographs are available. These are obtained using the District's method of routing generated synthetic hydrographs for rural and urban areas.[3/]

It is assumed in the routing operations that the flows are contained completely in the channel, i.e., no overbank flooding was occurring.

TABLE 5
SAN FRANCISQUITO Creek
1% FLOWS

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	3780.	3178.	1913.	691.	12.24	Bear Creek U/S Confluence
ROUTED TO	2	3780.	3178.	1913.	691.	12.24	
HYDROGRAPH AT	2	4070.	3503.	2123.	768.	14.63	San Fran. Inflow to Lake Searsville
ROUTED TO	2	3559.	3282.	2028.	740.	14.63	San Fran. Outflow from Lake Searsville
2 COMBINED	4	5600.	5277.	3263.	1201.	26.87	San Fran. D/S Bear Cr. Confluence
ROUTED TO	4	5517.	5182.	3249.	1201.	26.87	
HYDROGRAPH AT	4	1150.	804.	440.	157.	3.26	
ROUTED TO	4	1150.	304.	440.	157.	3.26	
2 COMBINED	4	6700.	6139.	3855.	1440.	30.13	San Fran. U/S of Los Trancos Confluence
ROUTED TO	5	6700.	6139.	3855.	1440.	30.13	
HYDROGRAPH AT	3	2330.	1766.	1044.	379.	7.25	Los Trancos U/S of Confluence
ROUTED TO	3	2330.	1766.	1044.	379.	7.25	
2 COMBINED	3	9030.	7859.	4871.	1819.	37.38	
HYDROGRAPH AT	5	90.	44.	22.	8.	0.33	
ROUTED TO	5	90.	44.	22.	8.	0.33	
2 COMBINED	5	8600.	7877.	4915.	1836.	37.71	San Fran. @ U.S.G.S. Sta. #1645
ROUTED TO	6	8394.	7835.	4914.	1835.	37.71	
HYDROGRAPH AT	6	650.	408.	219.	79.	1.74	
ROUTED TO	6	650.	408.	219.	79.	1.74	
HYDROGRAPH AT	6	370.	259.	160.	56.	1.00	
HYDROGRAPH AT	6	133.	92.	51.	18.	0.43	
2 COMBINED	6	503.	349.	211.	75.	1.43	
ROUTED TO	6	177.	177.	162.	75.	1.43	
3 COMBINED	6	9115.	8370.	5276.	1989.	40.88	San Fran. @ El Camino Real
HYDROGRAPH AT	7	288.	201.	124.	44.	0.81	
HYDROGRAPH AT	7	104.	72.	40.	14.	0.35	
2 COMBINED	7	392.	272.	164.	58.	1.16	
ROUTED TO	7	142.	141.	130.	58.	1.16	
2 COMBINED	7	9253.	8509.	5406.	2047.	42.04	San Fran. Near & D/S of Middlefield

TABLE I-6 , SAN FRANCISQUITO Creek
10% FLOWS.

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA	
HYDROGRAPH AT	1	2076.	1811.	967.	349.	12.24	Bear Creek U/S Confluence
ROUTED TO	2	2076.	1811.	967.	349.	12.24	
HYDROGRAPH AT	2	2253.	2030.	1091.	395.	14.63	San Fran. Inflow to Lake Sears
ROUTED TO	2	2034.	1864.	1011.	367.	14.63	San Fran. Outflow from Lake Se
2 COMBINED	4	3195.	3114.	1731.	639.	26.87	San Fran. D/S Bear Cr. Conflue
ROUTED TO	4	3174.	3039.	1725.	639.	26.87	
HYDROGRAPH AT	4	561.	425.	202.	73.	3.26	
ROUTED TO	4	561.	425.	202.	73.	3.26	
2 COMBINED	4	3620.	3375.	1944.	726.	30.13	San Fran. U/S of Los Trancos C
ROUTED TO	5	3620.	3375.	1944.	726.	30.13	
HYDROGRAPH AT	3	1200.	968.	505.	183.	7.25	Los Trancos U/S of Confluence
ROUTED TO	3	1200.	968.	505.	183.	7.25	
2 COMBINED	3	4820.	4309.	2435.	909.	37.38	
HYDROGRAPH AT	5	47.	23.	10.	4.	0.33	
ROUTED TO	5	47.	23.	10.	4.	0.33	
2 COMBINED	5	4412.	4170.	2375.	886.	37.71	San Fran. @ U.S.G.S. Sta. #164
ROUTED TO	6	4399.	4143.	2373.	886.	37.71	
HYDROGRAPH AT	6	301.	205.	94.	34.	1.74	
ROUTED TO	6	301.	205.	94.	34.	1.74	
HYDROGRAPH AT	6	246.	170.	103.	36.	1.00	
HYDROGRAPH AT	6	76.	48.	21.	8.	0.43	
2 COMBINED	6	321.	217.	124.	44.	1.43	
ROUTED TO	6	152.	151.	121.	44.	1.43	
3 COMBINED	6	4787.	4468.	2573.	964.	40.88	San Fran. @ El Camino Real
HYDROGRAPH AT	7	191.	132.	80.	28.	0.81	
HYDROGRAPH AT	7	59.	37.	16.	6.	0.35	
2 COMBINED	7	251.	169.	96.	34.	1.16	
ROUTED TO	7	123.	122.	95.	34.	1.16	
2 COMBINED	7	4909.	4589.	2660.	998.	42.04	San Fran. Near & D/S of Middlef

MATADERO-BARRON-ADOBE CREEKS

PALO ALTO FLOOD BASIN

HYDROLOGY

DETERMINATION OF DESIGN FLOWS

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GENERAL INFORMATION

Matadero, Barron, and Adobe Creeks are tributaries to the Palo Alto Flood Basin (see Figure 2-1). The total drainage area of the three creeks is nearly 28 square miles at the flood basin. Nearly half of the watershed area is on the valley floor and has undergone extensive urbanization. The watershed receives an average annual rainfall ranging from 13 inches in the flood basin to 34 inches in the uppermost reaches of Adobe Creek where the elevation is over 2,600 feet. On none of the three creeks are there reservoirs which function as a flood control facility.

The objective of this report is to document the hydrology work done to determine the design flows. It will discuss the use of regression equations and the analysis of recorded data, together with the special treatment of urban areas.

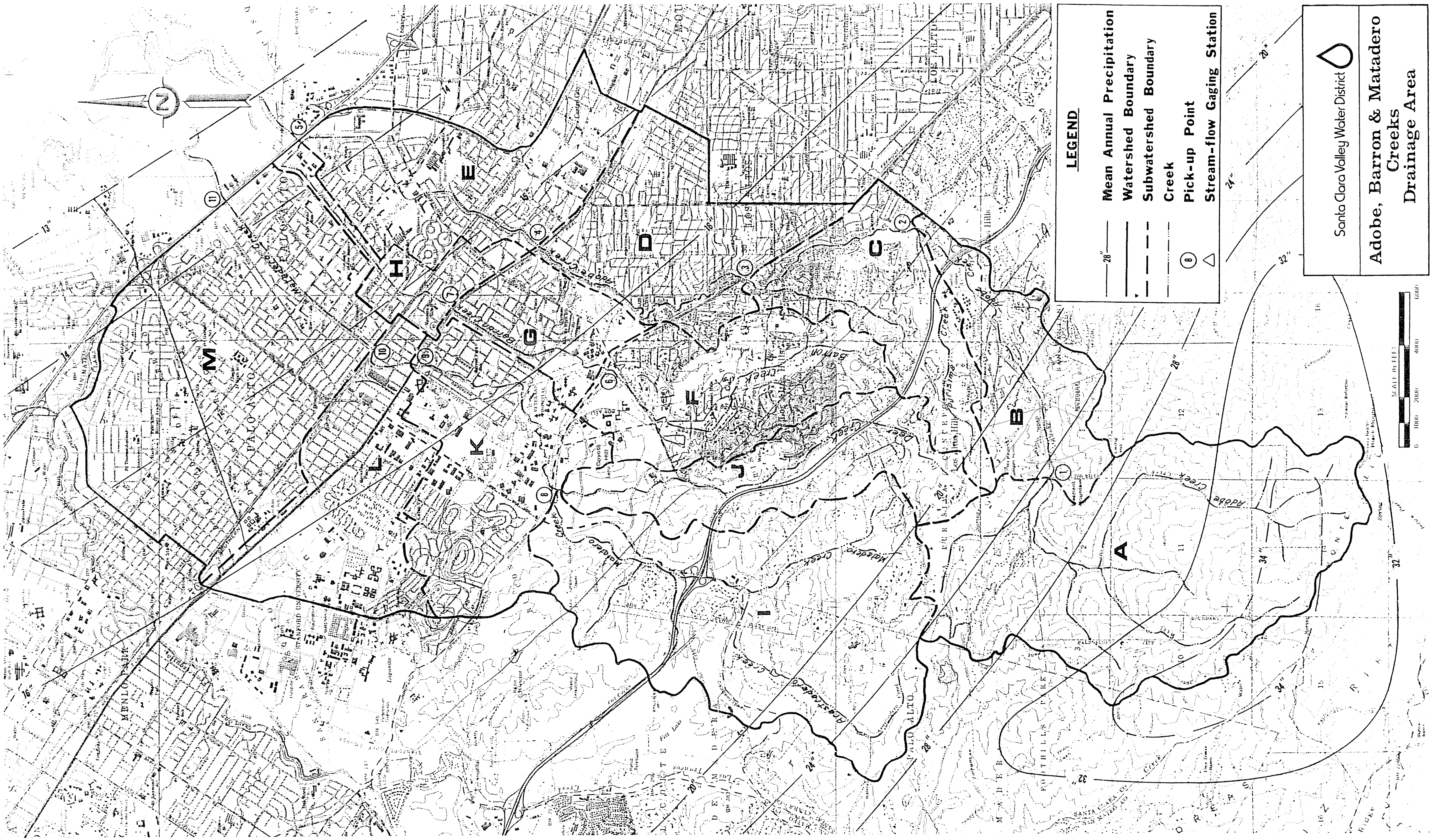
LEGEND
28" Mean Annual Precipitation
Watershed Boundary
Subwatershed Boundary
Creek
8 Pick-up Point
Stream-flow Gaging Station
Santa Clara Valley Water District
Adobe, Barron & Matadero Creeks Drainage Area
SCALE IN FEET
0 1000 2000 4000 6000
A
B
C
D
E
F
G
H
I
J
K
L
M
Adobe Creek
Barron Creek
Matadero Creek
Deer Creek
Arastradero Creek
Purisima Creek
13"
14"
16"
20"
24"
28"
32"
34"
PALO ALTO
MENLO PARK
STANFORD UNIVERSITY

Fig. 2-1

DISCUSSION OF PROCEDURES

Basin Characteristics

The Matadero, Adobe, and Barron Creeks Watersheds are each divided into two sections: a rural area, which includes all areas upstream of Foothill Expressway; and an urban area which includes all areas downstream. Rural subareas include: A, B, C, F, I, and J. Urban subareas include: D, E. G, H, K, L, and M (Figure 2-1). The subareas have the following basin characteristics:

Location	Sub-Area	Drainage Area* (Sq. Mi.)	M.A.P. (In.)	LLS	T_c (hrs)
Adobe Cr. @ City Limits	A	4.25	30	15.9	.6
Adobe Cr. U/S Purissima Creek	B	1.28	21	16	.67
Adobe Cr. @ Foothill Expwy.	C	1.57	18	36	.98
Adobe Cr. @ El Camino	D	Ai=1.08/ Ap=.72	16	-	Ti=.4/ Tp=2.4
Adobe Cr. U/S Barron Creek	E	Ai=1.16/ Ap=.78	14.5	-	Ti=.3/ Tp=2.3
Barron Cr. @ Foothill Expwy.	F	1.76	17	31	1.1
Barron Creek @ El Camino	G	Ai=.48/ Ap=.32	16	-	Ti=.4/ Tp=2.4
Barron Cr. U/S Adobe Cr.	H	Ai=.32/ Ap=.21	15	-	Ti=.4/ Tp=2.4
Matadero Cr. U/S Deer Creek	I	3.92	20	56	1.2
Deer Cr. U/S Matadero Creek	J	1.60	18	40	1.0
Matadero Cr. @ El Camino	K	Ai=.81/ 40% Ap=1.22 60%	16.5	-	Ti=.45/ Tp=2.45
Matadero Cr. @ Alma	L	Ai=1.12/ Ap=.75	16	-	Ti=.4/ Tp=2.4
Matadero Cr. @ Bayshore	M	Ai=2.49/ Ap=1.66	15	-	Ti=.3/ Tp=2.3

*Ai = impervious urban area; Ap= pervious urban area.

Urban areas are treated based on the District's method with a ratio of imperviousness approximating medium residential density.

Recorded Data and Application of Regional Regression Equations

The District's hydrology method includes a checking procedure that uses additional data wherever available. Streamflow records, stochastically generated data or rainfall data are all utilized. Following is a discussion of all of these procedures and their application to Matadero, Barron, and Adobe Creeks.

Streamflow Record

There is only one stream gage station in the nearly 28 square mile drainage area encompassed by Matadero-Barron and Adobe Creeks. The one station (U.S.G.S. Station #1660 located on Matadero Creek downstream of El Camino Real) has been operating since 1952 and was used as one of the base stations in the regional frequency regression studies.

Analysis of the 24 years of maximum annual recorded streamflows produced the following statistics:

N = 24 Years
M = 2.436
6 = .454
g = -.6 (regional skew)
1% = 1,945 cfs

The skew coefficient is difficult to determine reliably for stream gages which have a short period of record. The Water Resources Council recommends for $N \leq 25$ to use a regional skew (-.6).

Comparisons with Similar Watersheds

The length and reliability of the records at the Matadero Creek gage have enabled it to be used as a base station in the Regional Regression Analysis (Reference 4). The 23 years of available record (1953-1975) were correlated in a matrix with data from 20 other streamflow stations with up to 60 years of record. The correlation enhanced the Matadero gage statistics to be equivalent to 56 years of record. Recorded and estimated annual maximums are compiled in Table 2-1. Using the mean and standard deviation calculated from the 20-station/60-year recorded and synthetic data matrix, and the regional skewness of -.6*, the 1% flood peak was estimated to be 2,176 cfs. The log Pearson Type III plotting is shown on Figure 2-1.

Flood Estimates from Precipitation

Flood discharge flows in streams are affected by the combinations of prior rainfall, soil condition, and available storage capacity. Under proper conditions, a storm of a certain intensity may cause a flood or larger size than what is expected. This is obvious as the dates of the maximum recorded one-day* rainfalls, compiled in Table 2-2 for several precipitation stations in the Matadero-Barron-Adobe Creeks watersheds not necessarily fall with the days of the larger floods. Maximum one day rainfall was used as it was readily available. The maximum 24-hour rainfall would be approximately 13% higher.

---

* The regional coefficient of skewness is used herein rather than the skew obtained from stochastic data, because the later skew does not necessarily reflect more information than the former. Loss of information could happen as well.

TABLE 2-1

Maximum Annual Peak Flows 1915*-1976
U.S.G.S. Gage Matadero Creek @ Palo Alto

Year	Peak
1915	45. Est.*
1916	672. Est.
1917	353. Est.
1918	641. Est.
1919	136. Est.
1920	271. Est.
1921	317. Est.
1922	912. Est.
1923	84. Est.
1924	582. Est.
1925	55. Est.
1926	363. Est.
1927	902. Est.
1928	891. Est.
1929	345. Est.
1930	507. Est.
1931	46. Est.
1932	156. Est.
1933	294. Est.
1934	181. Est.
1935	672. Est.
1936	397. Est.
1937	873. Est.
1938	486. Est.
1939	39. Est.
1940	671. Est.
1941	505. Est.
1942	295. Est.
1943	1240. Est.
1944	553. Est.
1945	1256. Est.
1946	254. Est.
1947	20. Est.
1948	104. Est.
1949	44. Est.
1950	317. Est.
1951	362. Est.
1952	765. Est.
1953	535.
1954	26.
1955	170.
1956	854.
1957	28.
1958	672.
1959	340.
1960	139.
1961	45.
1962	365.
1963	641.
1964	223.
1965	219.
1966	311.
1967	765.
1968	490.
1969	792.
1970	368.
1971	388.
1972	170.
1973	1100.
1974	570.
1975	272.

*Peak flow rates for 1915-1953 were estimated using multiple cross correlation with surrounding stations as part of 1976 Regional Regression Study.

TABLE 2-2

Records of Rainfall
Matadero-Barron-Adobe Creeks

		Max. Annual Recorded 1-Day Rainfall		
# 3	Palo Alto City Hall	1.	2.77	December 22, 1955
	Elev. - 23	2.	2.42	January 21, 1967
	1954-1976	3.	2.25	September 18, 1960
#64	Mt. View (Lenihan)	1.	2.33	January 22, 1967
	Elev. - 100	2.	2.31	December 1, 1973
	1964-1976	3.	2.28	November 29, 1970
#24	Dahl Ranch	1.	6.00	January 21, 1967
	Elev. - 1680	2.	3.40	January 30, 1968
	1965-1976	3.	3.40	November 28, 1970

The mean annual precipitation (M.A.P.) reflects well the integrated effects of availability of atmospheric moisture, of elevation and topography, and of efficiency of the orographic mechanism, which are significant parameters of storm rainfall. It was, therefore, possible to develop a relationship of average storm values vs. M.A.P. for different durations and frequency using data collected from rainfall stations in and around the county. The 24-hour rain with an exceedance frequency of 1% is used as the design storm.

The unit hydrograph theory was employed to convert rainfall to runoff. Clark's synthetic unit hydrograph was used with the Corps of Engineers' 24-hour standard storm distribution and a 30% loss rate. It has been found that a 30% loss rate applied to the one percent storm, closely approximates the one percent flood on adjacent watersheds. The results of this procedure are compiled in the flow summary, Table 2-3. Peak 1% flow rate at the U.S.G.S. gaging station is 2,091 cfs using this approach. These are rough figures and a parameter optimization technique would be necessary to refine them further; however, they do provide a guide to prevent gross errors.

TABLE 2-3

1% Rainfall Generated Flows
Matadero-Barron-Adobe Creeks

		Peak	6-Hour	24-Hour	Area	
Adobe Creek @ City Limits	1	1637	1195	710	4.25	
Adobe Creek U/S Purissima Creek	2	2117	1495	880	5.53	
Adobe Creek @ Foothill Expressway	3	2685	1834	1071	7.10	
Adobe Creek @ El Camino	4	2900	2037	1260	8.90	
Adobe Creek U/S Barron Creek	5	3081	2253	1463	10.84	
Barron Creek @ Foothill Expressway	6	574	368	206	1.76	
Barron Creek @ El Camino	7	665	457	290	2.56	
Barron Creek U/S Adobe Creek	5	712	516	346	3.09	
Adobe-Barron @ Bayshore Freeway	5	3793	2768	1809	13.93	
Matadero Creek U/S Deer Creek	8	1295	892	506	3.92	
Deer Creek U/S Matadero Creek	8	575	353	195	1.60	
Matadero Creek @ Foothill Expressway	8	1870	1238	701	5.52	
Matadero Creek @ El Camino	9	2091	1465	909	7.55	U.S.G.S.
Matadero Creek @ Alma Street	10	2300	1676	1110	9.42	
Matadero Creek @ Bayshore Freeway	11	2741	2145	1554	13.57	
Matadero-Barron-Adobe Inflow to Palo Alto Flood Basin	5,11	6534	4913	3360	27.50	

OTHER STUDIES - PAST AND PRESENT

The hydrology of Matadero-Barron and Adobe Creeks has been studied at different points several times in the past. Values listed in the 1972 Manual of Design Flows are a conglomeration of application of several versions of regional regression equations.

The past regional equations, as developed in 1972 (3rd generation), have been used until revised by the 1976 (4th generation) equations. How well each set of equations represents the tributaries to the Palo Alto Flood Basin is shown graphically in Figure 2-2 where each flow frequency curve is plotted in comparison with the recorded data at the Matadero stream gage. The U. S. Geological Survey has developed its own set of regression equations for the San Francisco Bay Region. These equations were used to develop the U.S.G.S. flow frequency curve in Figure 2-2.

September 1976

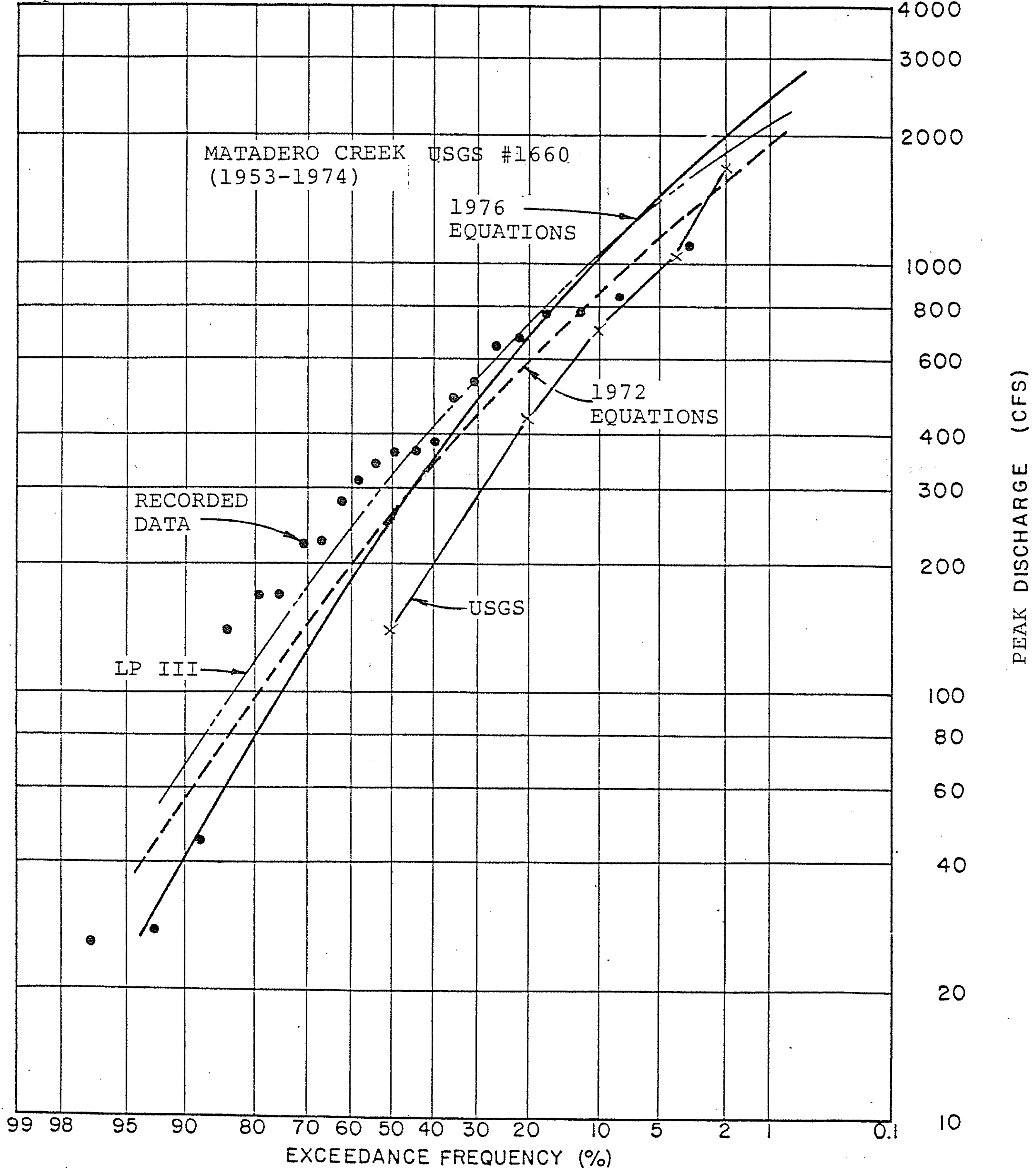

FIGURE 2-2

CONCLUSIONS

This report has summarized the work completed on the Matadero, Barron, and Adobe Creeks hydrology.

The flow rates have been computed from the 1976 Regional Regression Equations; checked against statistical analysis of systematic streamflow records; compared with hydrologically similar watersheds; checked with data generated from rainfall; and contrasted with past studies of our own and others.

Table 4 is a summary of results obtained from the various analyses done at the site of the U.S.G.S. Station, on Matadero Creek at Palo Alto.

TABLE 2-4

SUMMARY OF FLOW RESULTS @ GAGED LOCATION
REGIONAL SKEW = -.60 *

Method Used	Mean M	$N_{e,m}$	σ	$N_{e,\sigma}$	1% cfs	10% cfs
1) Statistical Analysis of Recorded Data	2.436	24	.454	24	1945	956
2) Statistical Analysis of Recorded and Estimated Data	2.456	57	.469	57	2176	1070
3) Rainfall Data	-	-	-	-	2091	-
4) Regional Equation Alone		12			2359	1180
5) Weighted Values (2 & 4)					2200	1070

*According to the W.R.C. guidelines the average skew is equal to -.6 since the number of years of record are less than 25 years.

It is apparent, therefore, that the weighted results do not differ appreciably from the regional ones. In fact, all of the results obtained from Item 2 through 5 of above table are within 10% of each other. Because the statistical values are based partially on recorded data which are indirectly biased by urbanization and upstream restriction, and because of homogeneity with other non instrumented locations, the results obtained from the regional equations and the application of the urban model have been chosen and used. The recommended 1% and 10% design flows are summarized in Tables 2-5 and 2-6. respectively. The values in these tables should be rounded up to significant figures.

TABLE 2-5

Recommended Design Flows - 1% Flood
Matadero-Barron-Adobe Creeks

		Peak	6-Hour	24-Hour	Area	
Adobe Creek @ City Limits	1	1558	1015	597	4.25	
Adobe Creek U/S Purissima Creek	2	2002	1330	774	5.53	
Adobe Creek @ Foothill Expressway	3	2748	1707	983	7.10	
Adobe Creek @ El Camino	4	2938	1910	1171	8.90	
Adobe Creek U/S Barron Creek	5	3080	2126	1375	10.84	
Barron Creek @ Foothill Expressway	6	762	403	222	1.76	
Barron Creek @ El Camino	7	829	492	305	2.56	
Barron Creek U/S Adobe Creek	5	834	552	362	3.09	
Adobe-Barron @ Bayshore Freeway	5	3914	2676	1736	13.93	
Matadero Creek U/S Deer Creek	8	1455	873	489	3.92	
Deer Creek U/S Matadero Creek	8	744	394	215	1.60	
Matadero Creek @ Foothill Expressway	8	2199	1260	704	5.52	
Matadero Creek @ El Camino	9	2359(2%-2012)	1486	912	7.55	U.S.G.S.
Matadero Creek @ Alma Street	10	2545(2%-2207)	1698	1113	9.42	
Matadero Creek @ Bayshore Freeway	11	2928	2166	1557	13.57	
Matadero-Barron-Adobe Inflow to Palo Alto Flood Basin	5,11	6842	4842	3291	27.50	

TABLE 2-6

Ten Year Flood Flows
Matadero-Barron-Adobe Creeks

		Peak	6-Hour	24-Hour	Area
Adobe Creek @ City Limits	1	821	615	316	4.25
Adobe Creek U/S Purissima Creek	2	1111	788	399	5.53
Adobe Creek @ Foothill Expressway	3	1433	977	490	7.10
Adobe Creek @ El Camino	4	1635	1161	633	8.90
Adobe Creek U/S Barron Creek	5	1822	1360	778	10.84
Barron Creek @ Foothill Expressway	6	323	199	95	1.76
Barron Creek @ El Camino	7	409	280	159	2.56
Barron Creek U/S Adobe Creek	5	455	334	199	3.09
Adobe-Barron @ Bayshore Freeway	5	2277	1694	977	13.93
Matadero Creek U/S Deer Creek	8	641	445	218	3.92
Deer Creek U/S Matadero Creek	8	320	198	93	1.60
Matadero Creek @ Foothill Expressway	8	961	639	311	5.52
Matadero Creek @ El Camino	9	1181	844	459	7.55
Matadero Creek @ Alma Street	10	1378	1037	608	9.42
Matadero Creek @ Bayshore Freeway	11	1794	1465	928	13.57
Matadero-Barron-Adobe Inflow to Palo Alto Flood Basin	5,11	4071	3158	1904	27.50

STEVENS AND PERMANENTE CREEKS HYDROLOGY

DETERMINATION OF DESIGN FLOWS

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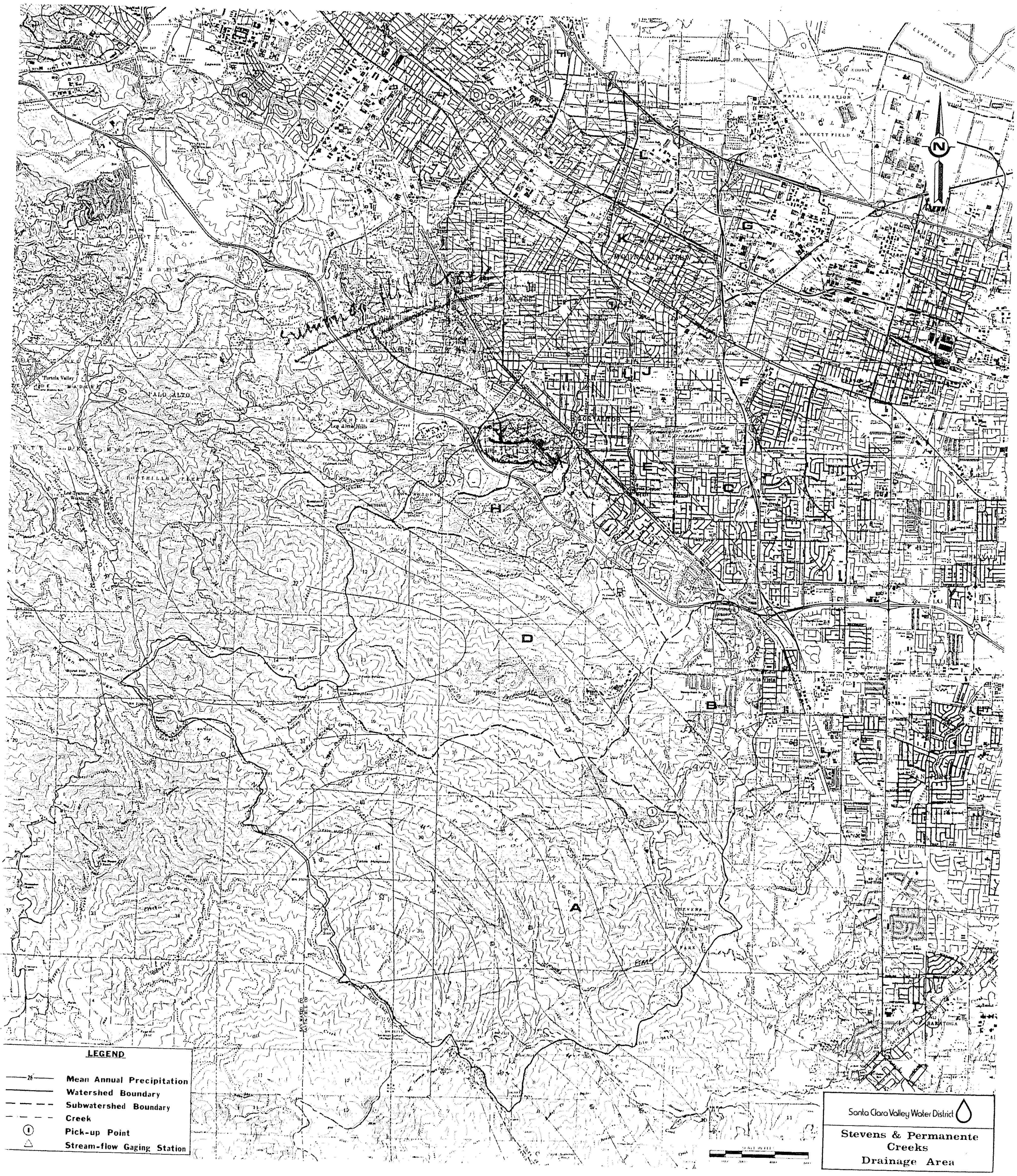
GENERAL INFORMATION

The Stevens Creek watershed lies on the northeastern slopes of the Santa Cruz Mountains in Santa Clara County between Permanente Creek and Calabazas Creek. As shown in Figure 3-1, Stevens Creek runs easterly from its headwaters till Stevens Creek Reservoir. Downstream of the reservoir the creek runs northerly through Sunnyvale and Mountain View. It discharges into South San Francisco Bay through Whisman Slough just to the east of Permanente Creek. The total drainage area of the watershed upstream of Highway 101 is 38 square miles.

In addition to various concrete culverts and channel modifications, the flow on the creek is affected by the presence of Stevens Creek Reservoir with a 3,600 acre-feet capacity. In 1960 a flood control diversion structure was constructed to divert a portion of high flows from Upper Penitencia Creek to Stevens Creek. Because of that the part of Permanente Creek upstream of the diversion is considered as a part of the Stevens Creek watershed and included in the calculation of design flows. The hydrology of the lower part of Permanente is analyzed separately and discussed in the last chapter of this report.

The mean annual precipitation on the basin varies from a high of about 34 inches on the upper slopes of the Santa Cruz Mountains, to a low of about 13 inches on the valley floor near where the stream discharges into the Bay. The majority of this precipitation occurs in the rainy season between November and April.



Santa Clara Valley Water District

Stevens & Permanente Creeks Drainage Area

The watershed's elevation ranges from 2,300 feet, down to near sea level. The upper elevations are characterized by moderately dense forests and steep slopes. The valley floor is marked by moderate to high urbanization and a gentle slope towards the Bay.

DISCUSSION OF PROCEDURES

Basin Characteristics

The Stevens and Permanente Creeks Watershed is divided into two major watersheds, rural which includes subareas A, B, D, and H; and urban, which includes subareas C, E, F, G, I, J, K, and L (Figure 3-1). The subareas have the following basin characteristics:

Location	Sub Area	Drainage* Area (Sq. Mi.)	M.A.P. (In.)	Lls $(mi^2/ft/ft^{1/2})$	T_c (Hrs)
Stevens Crk U/S Res	A	17.30	35.5	300	2.3
Stevens Crk @ I-280	B	2.90	20.0	62	1.5
North Branch Permanente Crk @ I-280	D_1	3.62	26.0	61	0.5
Hale Crk @ SCVWD #33	H	2.62	18.0	37	1.0
Stevens Crk U/S Diversion	C	Ai=2.54 Ap=1.89	16.0	-	.5 2.3
Permanente Diversion U/S Stevens Crk	E	Ai= .68 Ap= .46	16.0	-	.5 2.3
Stevens Crk @ El Camino Real	F	Ai= .91 Ap= .60	15.0	-	.5 2.3
Stevens Crk @ U.S. 101	G	Ai=1.76 Ap=1.18	14.0	-	.5 2.3

*Ai = impervious urban area; Ap = pervious urban area

In rural areas, the preliminary evaluation of design flows from regional regression equations is determined using the drainage areas, mean annual precipitation and basin factors. These preliminary values are compared with peak flows obtained from rainfall by employing the unit hydrograph theory and HEC-1 program. Although this latter technique is used mainly to define design hydrographs, the resultant peak flow is compared and balanced against the regional one. As will be shown later, the end result of above procedure is further compared with results from the site specific analysis of recorded data if available.

Urban areas are treated based on the District's method of urban hydrology. The density of land use is determined by the ratio of impervious to pervious parts.

Recorded Data and Application of Regional Equations

Inherent in the District's hydrology method are checking procedures which use additional data whenever possible. Annual peak flow records from stream gaging stations on Stevens Creek and Upper Permanente Creek were subjected to a statistical analysis in order to determine the frequency regime. The values obtained in this manner were compared with the flows generated by the regional equations. In Stevens and Upper Permanente Watersheds, there are three gaging stations. The following is a discussion of the analysis of the observed data from these stations.

Streamflow Station No. 44

Station No. 44, immediately below the reservoir (17.5 square miles drainage area) has a 45-year record but only 6 years are before the dam was built. These 6 years (1930 through 1936) were separated from the record, and the statistics for this pre-dam period were calculated. The statistics were not found to represent the frequency regime of Stevens Creek for the mean or for standard deviation. Many analyses were attempted using the data after the dam was built. Knowing that this data represents regulated flows, with no reliable data to further generate natural flows, and knowing that those regulated flows are based on nonrandom events, no frequency regime was adopted.

Streamflow Station No. 32

Streamflow Station No. 32 on Permanente Creek just upstream of the Permanente Diversion has a data record of 31 years, from 1945 through 1975. A frequency analysis of this record yields a one percent flow of 287 cfs. The recorded data were determined to be inconsistent with flows obtained using the District's procedures of regional regression and urban hydrologic analysis.

Streamflow Station No. 35

The second station downstream of the reservoir, Stevens Creek at SPRR, has 35 years of record (1938-1975), all of it since Stevens Creek Reservoir was built. However, the drainage area of this station (25 square miles), is larger than the area tributary to the Station 44, and it is more than 7 miles downstream. For these reasons, it is thought that the effect of the reservoir on the data from this station is less significant than that on data from Station No. 44.

The other effect on the data from Station No. 35 is that of the Permanente Diversion. Constructed in 1959, the diversion was designed to shift a portion of flood flows from the nine square mile Upper Permanente Creek Watershed to Stevens Creek. The diversion passes flows less than 150 cfs downstream to Lower Permanente Creek. Flood flows greater than 150 cfs are partially diverted to Stevens Creek. The diversion passes a maximum capacity of 1,700 cfs to Stevens Creek, at which stage the contribution of the diversion to Lower Permanente is 500 cfs. Thus, Upper Permanente Creek acts as a tributary to the Stevens Creek system, supplying a maximum of 1,700 cfs. An inspection of data from Station No. 32, on Permanente Creek upstream of the

Diversion, shows only 1 out of 36 years of record where the maximum flow was greater than the 150 cfs necessary to cause diversion to Stevens Creek. While the design of the Permanente Diversion provides that only flows in excess of 150 cfs are diverted to Stevens Creek, from actual experience, the narrow "window" in the concrete floor of the channel frequently clogs with debris forcing a large portion of the 0 to 150 cfs flows on to Stevens Creek.

The statistical analysis of the data from Station No. 35 is shown in Appendix 1. The one percent flow obtained from this data is 8,900 cfs. Because of all of the above inaccuracies, this value was not used for modifying regional values. It was only used for comparison.

At Station No. 35, Stevens Creek at SPRR, routing the regional values yielded a one percent flow of 8,300 cfs. This is close to the statistically derived value of 8,900 cfs except that the regional value includes a 1,700 cfs contribution from the Permanente Diversion. The statistical value does not reflect this condition, making it that much more conservative. In other words, the statistical analysis at that station shows that the regional values are on the lower side.

Initial Reservoir Level - Coincident Frequency

Stevens Creek Reservoir, constructed in 1936, is operated as a water conservation facility rather than for flood control. It does, however, provide some flood protection since it could be less than full when a storm occurs. Also, even a full reservoir will attenuate a flood peak to some extent.

An incidental initial storage analysis was done on Stevens Creek Reservoir in order to determine the most likely initial storage for use with the design flow calculations. The Storage-Duration-Frequency Graphs for Stevens Creek (Curves K, L, and M in Figure 3-2) show that the historical trend is towards the reservoir being full a greater percentage of the time. The storage-frequency curve representing the entire period of operation (Curve L) was chosen as a basis for the coincidental frequency analysis.

This curve was used to obtain four initial reservoir levels (A, B, C, and D) as presented in Table 3-1. The levels correspond to points on Curve L where the reservoir volume is equaled or exceeded 10%, 25%, 50% and 75% of the time. Table 3-1 shows the results of routing various floods through the reservoir with these four initial storage conditions. At the one percent flood level, varying the initial storage has little effect on the maximum outflow. The use of Curve M (the most recent reservoir operation trend) would result in the initial storage level having an even smaller effect upon maximum outflow. Therefore, the full condition was chosen and used for design.

STORAGE DURATION CURVE

September, 1975

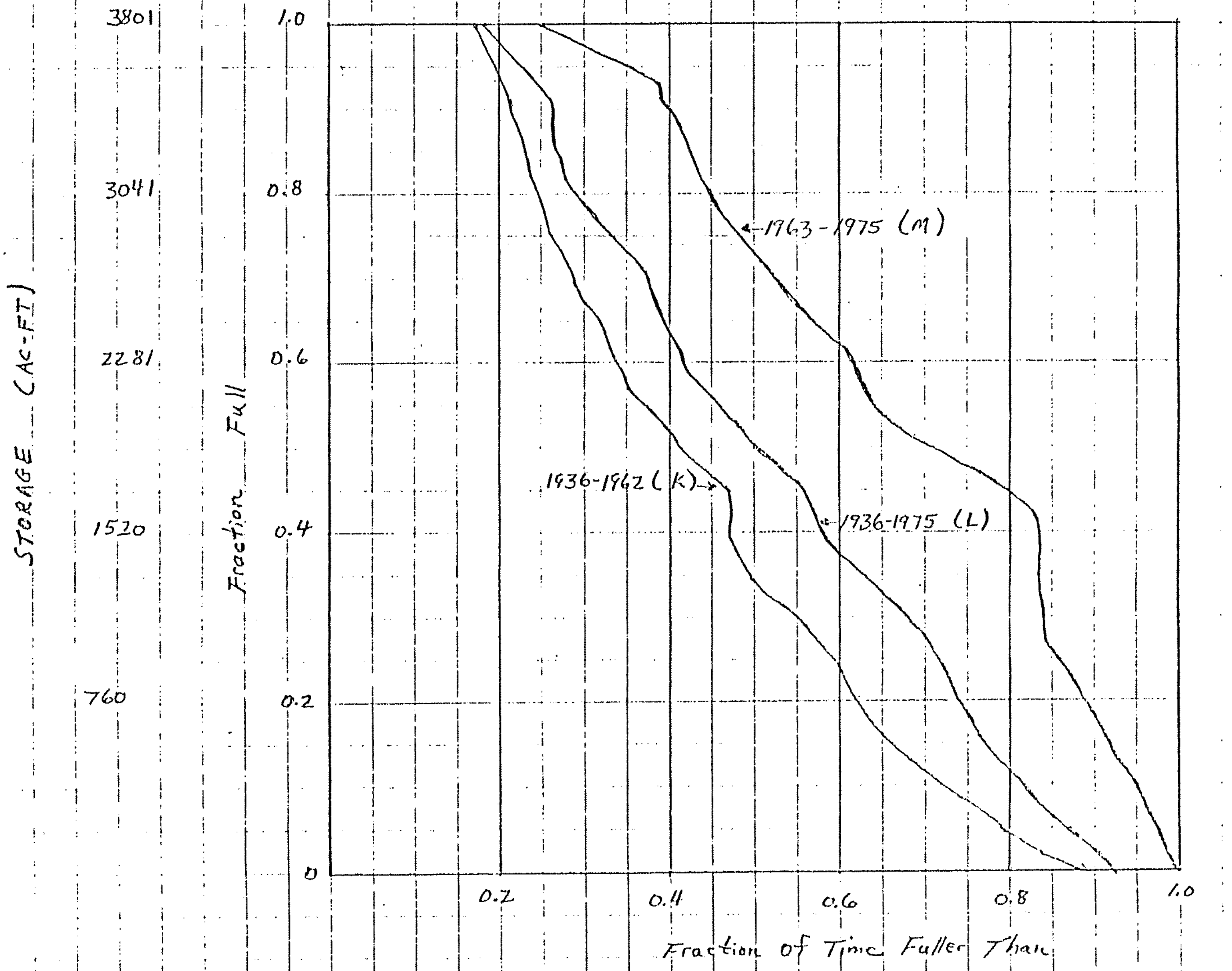


Figure 3-2

TABLE 3-1

Determination of Coincidental Initial Storage

Based on data through 1975, and based on the storage duration curve prepared for this period, four storages were chosen:

Storage Condition A = Initial Storage 3,600 A.F. (Full)

Storage Condition B = Initial Storage 3,250, exceeded 25% of the time

Storage Condition C = Initial Storage 1,750, exceeded 50% of the time.

Storage Condition D = Initial Storage 350, exceeded 75% of the time.

The following is a list of those storage conditions and the various results obtained after subjecting them to inflows of varying exceedance frequency:

Design Flow Exceedance Freq. (%)	Initial Storage (A.F.) Condition		MAX (I) cfs	MAX (O) cfs	Max Storage (E. of. P) A.F.
10	A	(3,600)	2,883	2,800	3,900
	B	(3,250)	2,887	2,762	4,116
	C	(1,750)	2,887	1,379	4,001
	D	(350)	2,887	-0-	3,157
2	A	(3,600)	4,793	4,413	4,226
	B	(3,250)	4,793	4,413	4,226
	C	(1,750)	4,793	4,049	4,202
	D	(350)	4,793	2,330	4,075
1	A	(3,600)	6,000	5,400	4,100
	B	(3,250)	5,900	5,247	4,280
	C	(1,750)	5,900	5,190	4,276
	D	(350)	5,900	3,573	4,169
.5	A	(3,600)	6,500	6,078	4,335
	B	(3,250)	6,500	6,078	4,335
	C	(1,750)	6,500	6,071	4,335
	D	(350)	6,500	5,132	4,279

PREVIOUS STUDIES

The hydrology of Stevens Creek has been studied several times in the past. The results of the most recent studies are shown in Table 3-2.

Pre 1972

In 1971 the District analyzed the hydrology of Stevens Creek to determine the one percent, two percent and five percent design flows (Reference 8). Similar to the present techniques, model hydrographs were developed from rainfall. Peaks and volumes of those hydrographs were balanced with results from the 1967 regional equations. The resultant hydrographs were combined and routed downstream. Full reservoir one percent design flows from that study compare quite closely with the 1976 computations for reaches downstream of the diversion.

1972 Regional Equations

In 1972, regional regression equations (third generation) were developed and used to calculate one percent design flows on Upper Stevens Creek. Routing and computational procedures were similar to those used in the 1976 study. The results can be seen in Table 3-2. The shift in the one percent design flows produced by the 1972 equations, versus those produced by the 1976 equations, is attributable to the utilization of an enlarged and updated data base in regressing the 1976 regional equations.

Experts' Analysis

The 1972 design flows on Stevens Creek were reviewed by a number of hydrologists and organizations. A panel of expert hydrologists was convened by the District in order to review the methods used. In addition to their overall acceptance of the 1972 methodology (Reference 9), they expressed their agreement with the design flows determined for Upper Stevens Creek (see Appendix 1).

In addition to the testimony of these experts, the 1972 flows on Stevens Creek were approved by Dames and Moore with regard to both method and result (see Appendix 1). They recommended the use of a one percent flow of 6,600 cfs at Highway 280, although they stated that this could "be on the high side by ten to twenty percent". As shown in Table 3-2, the 1976 design flow did prove to be ten percent lower at 5,970 cfs.

TABLE 3-2

COMPARATIVE 1% DESIGN FLOWS

FULL RESERVOIR

Location	Drainage Area (mi^2)	1971 Study (cfs)	1972 Equations (cfs)	1976 Equations (cfs)
Stevens Crk U/S Reservoir	17.30	6,680	7,560	6,000
Stevens Crk D/S Reservoir	17.30	6,300	6,890	5,460
Stevens Crk at Highway 280	20.02	-	7,580	5,970
Stevens Crk U/S Diversion	24.63	6,810	-	6,430
Permanente Crk Diversion to Stevens Crk	8.95	1,450	-	1,710
Stevens Crk D/S Diversion	33.58	8,260	-	7,860
Stevens Crk U/S Highway 101	38.03	8,690	-	8,330

LOWER PERMANENTE CREEK

General Information

Lower Permanente Creek (downstream of Permanente Diversion) lies almost entirely on the Santa Clara Valley floor with a maximum elevation of less than 1,000 feet. Consequently, mean annual rainfall for the entire basin is less than 16 inches and the watershed is quite urbanized. The watershed begins at the Permanente Diversion and has a drainage area of 8 square miles. The major tributary, Hale Creek, runs from the Santa Cruz foothills to approximately midway between the diversion and Highway 101.

1976 Design Flood Procedures

The one percent and ten percent design flows were calculated in the same manner as those for Stevens Creek, using the regional equations and urban hydrology. The computations were begun by using the undiverted balance of flows from Upper Permanente Creek as an inflow to Lower Permanente Creek with zero drainage area. This primary inflow was then routed downstream and combined with local hydrographs down to Highway 101.

Analysis of Basic Data

There is only one reliable station on Lower Permanente Creek: SCVWD No. 34, Permanente at California Street. This station has records from 1943 through 1964 when it was discontinued. This record is not considered to be affected by the diversion because no flows large enough to cause diversion to Stevens Creek were observed in the five years of record after the diversion was completed. An analysis of the data yields a one percent flow of 3,650 cfs. However, if this value is modified to reflect the effects of up to 1,700 cfs of diversion to Stevens Creek, the statistically derived value becomes 2,100 cfs which is close to the one percent design flow of 2,200 cfs calculated by the 1976 Regional Equations.

CONCLUSIONS

This report is the summary of work completed in the study of the hydrology of Stevens and Permanente Creeks. The application of the 1976 Regional Equations compares quite favorably with those developed by previous studies, and with an analysis of streamflow records, where appropriate.

After careful consideration, it is recommended that the flows derived using the 1976 Regional Regression Equations and full reservoir level be used for design purposes.

For Stevens Creek, the recommended one percent design flows are summarized in Table 3-3 along with the ten percent design flows in Table 3-4. For Permanente Creek, the one and ten percent design flows are summarized in Tables 3-5 and 3-6. The values in these tables should be rounded up to significant figures.

```
[illegible] HYDROLOGY [illegible]
RESULT [illegible] RURAL AREAS ARE [illegible] BA[illegible] 974 [illegible]
URBAN AREAS ARE TREATED ACCORDING TO SCVWD METHOD

                                 JOB SPECIFICATION
            NQ     NHR    NMIN   IDAY    IHR    IMIN  METRC   IPLT   IPRT  NSTAN
            50       0      30      0      0       0      0      0      5      0
                                   JOPER      NWT
                                       3        0

                              PEAK     6-HOUR    24-HOUR    72-HOUR     AREA
HYDROGRAPH AT         1      6000.      4620.      2658.      2552.    17.30  INFLOW TO RESERVOIR (AREA A)
ROUTED TO             1      5431.      4557.      2495.      2400.    17.30  OUTFLOW (RESERVOIR FULL=3804AF
ROUTED TO             2      5364.      4551.      2417.      2322.    17.30
HYDROGRAPH AT         2      1200.       684.       385.       370.     2.90
ROUTED TO             2      1200.       684.       385.       370.     2.90
 2 COMBINED           2      5944.      5163.      2802.      2692.    20.20  @ 280 CONFLUENCE
ROUTED TO             3      5945.      5166.      2780.      2670.    20.20
HYDROGRAPH AT         3      1148.       630.       389.       375.     2.54
HYDROGRAPH AT         3       579.       389.       213.       205.     1.89
 2 COMBINED           3      1652.      1007.       600.       580.     4.43
ROUTED TO             3       470.       467.       385.       371.     4.43
 2 COMBINED           3      6411.      5627.      3164.      3041.    24.63STEVENS U/S OF PERMANANTE DIV.
HYDROGRAPH AT         4      2566.      2136.      1231.      1182.     7.81  PERMA. U/S OF 280
ROUTED TO             3      2601.      2132.      1219.      1171.     7.81
HYDROGRAPH AT         3       307.       169.       104.       100.     0.68
HYDROGRAPH AT         3       141.        95.        52.        50.     0.46
 2 COMBINED           3       430.       260.       155.       150.     1.14
ROUTED TO             3       164.       163.       134.       129.     1.14
 2 COMBINED           3      2762.      2292.      1353.      1300.     8.95  PERMA. @ DIVERSION
ROUTED TO             3      1787.      1491.       829.       795.     8.95  DIVERTED FLOWS
 2 COMBINED           3      8163.      7090.      3993.      3837.    33.58  STEVENS D/S OF DIVERSION
ROUTED TO             5      8144.      7089.      3980.      3824.    33.58
HYDROGRAPH AT         5       395.       217.       134.       129.     0.91
HYDROGRAPH AT         5       176.       118.        65.        62.     0.60
 2 COMBINED           5       548.       331.       198.       191.     1.51
ROUTED TO             5       177.       176.       149.       144.     1.51
 2 COMBINED           5      8320.      7262.      4129.      3968.    35.09STEVENS @ EL-CAMINO
ROUTED TO             6      8320.      7258.      4105.      3944.    35.09
HYDROGRAPH AT         6       730.       400.       247.       238.     1.76
HYDROGRAPH AT         6       331.       223.       122.       117.     1.18
 2 COMBINED           6      1018.       616.       368.       355.     2.94
ROUTED TO             6       340.       338.       288.       278.     2.94
 2 COMBINED           6      8660.      7593.      4393.      4222.    38.03  STEVENS @ BAYSHORE
```

TABLE 3-3

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STEVENS CREEK HYDROLOGY
[ .T. JM AL EAS E E ANCE BAS ON 1976 AT
URBAN AREAS ARE TREATED ACCORDING TO SCVWD METHOD

                         JOB SPECIFICATION
          NQ   NHR  NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
          50     0    30     0     0     0     0     0     5     0
                         JOPER      NWT
                             3        0

                                PEAK    6-HOUR   24-HOUR   72-HOUR      AREA
 HYDROGRAPH AT            1    2883.     2615.     1368.     1314.     17.30  INFLOW TO RES. CRRE
 ROUTED TO                1    2842.     2552.     1260.     1210.     17.30  OUTFLOW (RES. FULL)
 ROUTED TO                2    2830.     2529.     1204.     1156.     17.30
 HYDROGRAPH AT            2     526.      336.      170.      163.      2.90
 ROUTED TO                2     526.      336.      170.      163.      2.90
  2 COMBINED              2    3083.     2806.     1374.     1319.     20.20  @ 280
 ROUTED TO                3    3090.     2804.     1358.     1304.     20.20
 HYDROGRAPH AT            3     745.      378.      209.      201.      2.54
 HYDROGRAPH AT            3     340.      206.       86.       83.      1.89
  2 COMBINED              3    1031.      576.      295.      284.      4.43
 ROUTED TO                3     381.      377.      269.      258.      4.43
  2 COMBINED              3    3471.     3180.     1626.     1562.     24.63  STEVENS U/S OF PERMANI
 HYDROGRAPH AT            4    1375.     1267.      667.      641.      7.81  PERMA. U/S OF 280
 ROUTED TO                3    1369.     1264.      659.      633.      7.81
 HYDROGRAPH AT            3     199.      101.       56.       54.      0.68
 HYDROGRAPH AT            3      83.       50.       21.       20.      0.46
  2 COMBINED              3     269.      149.       77.       74.      1.14
 ROUTED TO                3     152.      141.       76.       73.      1.14
  2 COMBINED              3    1519.     1404.      736.      706.      8.95  PERMA. @ DIVERSION
 ROUTED TO                3    1070.      995.      471.      452.      8.95  DIVERTED FLOWS
  2 COMBINED              3    4537.     4114.     2098.     2014.     33.58  STEVENS D/S OF DIVERS
 ROUTED TO                5    4548.     4115.     2088.     2005.     33.58
 HYDROGRAPH AT            5     256.      130.       72.       69.      0.91
 HYDROGRAPH AT            5     104.       63.       26.       25.      0.60
  2 COMBINED              5     343.      190.       98.       94.      1.51
 ROUTED TO                5     155.      153.       97.       93.      1.51
  2 COMBINED              5    4702.     4266.     2184.     2098.     35.09  STEVENS @ EL-CAMINO
 ROUTED TO                6    4697.     4269.     2165.     2079.     35.09
 HYDROGRAPH AT            6     474.      241.      134.      129.      1.76
 HYDROGRAPH AT            6     195.      118.       50.       48.      1.18
  2 COMBINED              6     639.      355.      183.      176.      2.94
 ROUTED TO                6     304.      300.      181.      174.      2.94
  2 COMBINED              6    5000.     4562.     2346.     2253.     38.03  STEVENS @ BAYSHORE
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TABLE 4

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 DESIGN FLOWS ON RURAL AREAS ARE BALANCED W/1976 REGIONAL RESULTS
 URBAN AREAS ARE TREATED BASED ON DISTRICT METHOD.

                               JOB SPECIFICATION
              NQ    NHR   NMIN   IDAY    IHR   IMIN METRC   IPLT   IPRT NSTAN
              50      1      0      0      0      0     0      0      5     0
                                JOPER        NWT
                                    3          0
                                 PEAK     6-HOUR    24-HOUR    72-HOUR        AREA
   HYDROGRAPH AT        7       1079.       626.       340.       173.        2.62 HALE CR. U/S FOOTHILL
   ROUTED TO            8        879.       620.       339.       173.        2.62
   HYDROGRAPH AT        8        469.       327.       202.        98.        1.32
   HYDROGRAPH AT        8        257.       180.        99.        48.        0.88
    2 COMBINED          8        688.       504.       301.       146.        2.20
   ROUTED TO            8        264.       263.       242.       146.        2.20
    2 COMBINED          8       1134.       872.       575.       319.        4.82 HALE CR. @ PERMANANTE
   HYDROGRAPH AT        0       1093.       730.       465.       249.        0.00 PERMA. DIVERSION CONTRIBUTION
   HYDROGRAPH AT        8        241.       168.       104.        50.        0.69
   HYDROGRAPH AT        8        132.        92.        51.        25.        0.46
    2 COMBINED          8        353.       259.       155.        75.        1.15
   ROUTED TO            8        137.       136.       126.        75.        1.15
    2 COMBINED          8       1225.       857.       586.       324.        1.15 PERMA. U/S OF HALE
    2 COMBINED          8       2359.      1700.      1160.       643.        5.97   "     D/S   "   "
   ROUTED TO            9       2108.      1694.      1158.       644.        5.97
   HYDROGRAPH AT        9        277.       193.       120.        58.        0.83
   HYDROGRAPH AT        9        151.       106.        58.        28.        0.55
    2 COMBINED          9        405.       297.       178.        86.        1.38
   ROUTED TO           10        162.       161.       149.        86.        1.38
    2 COMBINED          9       2269.      1850.      1306.       731.        7.35 PERMA. @ S.P.R.R.
   HYDROGRAPH AT       10        150.       105.        65.        31.        0.46
   HYDROGRAPH AT       10         80.        56.        31.        15.        0.30
    2 COMBINED         10        218.       160.        96.        46.        0.76
   ROUTED TO           10         88.        87.        81.        46.        0.76
    2 COMBINED         10       2356.      1936.      1386.       777.        8.11 PERMA. @ BAYSHORE
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TABLE A

TABLE - 3-5

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PERMAN  E & HALE CREEK HYDROLOGY (10 YEAR FLOO
DESIGN FLOWS ON RURAL AREAS ARE BALANCED W/1976 R  IONA RESULTS
URBAN AREAS ARE TREATED BASED ON DISTRICT METHOD.

                            JOB SPECIFICATION
          NQ   NHR  NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
          50     1     0     0     0     0     0     0     5     0
                          JOPER    NWT
                              3      0
                          PEAK   6-HOUR   24-HOUR   72-HOUR      AREA
  HYDROGRAPH AT      7     464.     301.     148.      75.       2.62 HALE CR. U/s FOOTHILL
  ROUTED TO          8     417.     297.     147.      75.       2.62
  HYDROGRAPH AT      8     290.     193.     109.      53.       1.32
  HYDROGRAPH AT      8     146.      93.      40.      19.       0.88
   2 COMBINED        8     409.     285.     149.      72.       2.20
  ROUTED TO          8     229.     226.     148.      72.       2.20
   2 COMBINED        8     644.     521.     295.     147.       4.82 HALE CR. @ PERMANANTE
  HYDROGRAPH AT      0     482.     376.     235.     117.       0.00 PERMA. DIVERSION CONTRIBUTION
  HYDROGRAPH AT      8     149.      99.      56.      27.       0.69
  HYDROGRAPH AT      8      75.      48.      21.      10.       0.46
   2 COMBINED        8     210.     146.      77.      37.       1.15
  ROUTED TO          8     119.     118.      76.      37.       1.15
   2 COMBINED        8     598.     486.     311.     154.       1.15 PERMA. U/s OF HALE
   2 COMBINED        8    1238.     993.     606.     301.       5.97   "    D/s "  "
  ROUTED TO          9    1168.     985.     604.     301.       5.97
  HYDROGRAPH AT      9     171.     114.      65.      31.       0.83
  HYDROGRAPH AT      9      85.      55.      24.      11.       0.55
   2 COMBINED        9     241.     168.      88.      42.       1.38
  ROUTED TO         10     143.     141.      88.      43.       1.38
   2 COMBINED        9    1309.    1126.     691.     344.       7.35 PERMA. @ S.P.R.R.
  HYDROGRAPH AT     10      93.      62.      35.      17.       0.46
  HYDROGRAPH AT     10      46.      29.      13.       6.       0.30
   2 COMBINED       10     130.      91.      48.      23.       0.76
  ROUTED TO         10      78.      77.      47.      23.       0.76
   2 COMBINED       10    1387.    1203.     738.     367.       8.11 PERMANANTE @ BAYSHORE
```

✓ Table 2

T.A 5-26

APPENDIX - 1

STEVENS CREEK HYDROLOGY FILE

(AVAILABLE FROM RESOURCES MANAGEMENT
DIVISION UPON REQUEST)

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